513

VOLUME 80 SEPARATE No. 513

PROCEEDINGS

AMERICAN SOCIETY
OF
CIVIL ENGINEERS

OCTOBER, 1954

FOUNDATION CONDITIONS IN THE CUYAHOGA RIVER VALLEY

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SOIL MECHANICS AND FOUNDATIONS DIVISION

{Discussion open until February 1, 1955}

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Printed in the United States of America

Headquarters of the Society
33 W. 39th St.
New York 18, N. Y.

PRICE $0.50 PER COPY

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This paper was published at 1745 S. State Street, Ann Arbor, Mich., by the American Society of Civil Engineers. Editorial and General Offices are at 33 West Thirty-ninth Street, New York 18, N. Y.

FOUNDATION CONDITIONS IN THE CUYAHOGA RIVER VALLEY

Ralph B. Peck,[1] M. ASCE

INTRODUCTION

One of the most significant contributions of soil mechanics to foundation engineering during the past 30 years has been the development of techniques for accurately describing the essential characteristics of deposits of soil. Without this information, no records of construction experience can be utilized to the extent of their potential value and no substantial progress in foundation engineering can be made.

The description of the essential characteristics of any subsurface deposit includes the results of borings and soil tests and a statement of the significant features of the geology. It also contains a digest of what is known concerning the behavior of the mass of soil as a whole, such as the results of field tests, of settlement observations, and of observations on the stability of structures.

This paper is a summary of foundation conditions in a portion of the Cuyahoga River valley located within the city of Cleveland, Ohio. It is an example of several regional studies that have been made since the advent of soil mechanics. It may be considered representative of recent progress in one important aspect of foundation engineering.

Moreover, the city of Cleveland and the Cuyahoga River valley are in themselves of exceptional interest to the foundation engineer. The portion of the valley discussed in this paper is one of the most heavily industrialized regions in the nation. It contains several steel mills, refineries, and manufacturing plants. Because the walls of the valley constrict the industrial area, there is a great concentration of heavily loaded structures and of railways, bridges, and viaducts. Hence, a summary of foundation experience in the valley represents knowledge immediately applicable in engineering practice.

Finally, the valley is of interest because of the complexity of subsurface conditions and the necessarily close relationship between geology and civil engineering. This relationship will become apparent in the discussion of subsurface characteristics.

General Description of Subsurface Conditions

In general, the city of Cleveland, Figure 1, is located on an upland at an elevation at least 660 ft above sea level. The elevation of Lake Erie is approximately 573 ft. Much of the upland is underlain at shallow depths by bedrock formations, primarily of sandstone and shale.

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However, in and near the downtown section the upland is a relatively flat plain developed upon sediments deposited by the forerunner of the Cuyahoga River.

The river itself follows a winding path in a generally northward direction through the city. It has eroded a valley of moderate width bounded by steep bluffs. The floor of this valley, represented by the stippled area in Figure 1, is at or below El. 600.

An east-west cross section showing the subsurface conditions (Sec. E-W, Figure 1) is represented by Figure 2. This figure demonstrates that the existing valley is a relatively minor depression of the ground surface compared to a much more impressive valley in the bedrock surface. Indeed, the bottom of the bedrock valley is probably located below present sea level. The approximate eastern and western boundaries of the bedrock valley as well as the approximate location of its deepest portion are shown in Figure 1. The width of the valley is at least 5 miles.

Much of the bedrock valley is filled with deposits of clay till and of glacial lacustrine clay or silty clay. These deposits extend upward to about El. 560. They are overlain by sand and silty sand.

Geological studies have indicated that the bedrock valley was the course of an important pre-glacial stream that flowed in a north-south direction. The northern extension of the stream is probably concealed in the basin of present Lake Erie. During several of the glacial advances from the north the ice extended into the valley and enlarged it. Subsequently, it deposited some of the till sheets now found in the subsoil. In addition, water was often ponded between the ice and the higher ground to the south, and glacial lakes were created in which lacustrine deposits were formed. The lacustrine clays and the till deposits are intermingled in heterogeneous fashion.

Near the close of the glacial epoch the water on several occasions stood at a considerably higher level than that of present Lake Erie. The highest of these levels is believed to have been at El. 812. The Cuyahoga River, flowing from south to north, brought coarse-grained sediments into the lake and deposited a substantial delta over the area now occupied by the central part of the city.

At various times subsequent to the maximum elevation of the lake, the water stood at different levels, some lower and some higher than that of Lake Erie. During the low-water periods the Cuyahoga River cut its present valley from the delta by removing the material between the existing bluffs. At the same time it helped to remove the material eroded from the top of the delta deposit and thus lowered the general level of the surrounding upland. During the high-water periods the river aggraded its valley floor with silty, sometimes organic materials.

Thus, at the present time the Cuyahoga River is a relatively insignificant stream flowing in a small valley which it has cut from sand deposits laid down during the latter part of the glacial epoch by a much more important northward flowing river. The glacial river in turn was itself the successor to the much more ancient large river that carved the great bedrock valley underlying much of the city of Cleveland.

In this paper attention will be concentrated on the small area outlined in Figure 1 and shown in greater detail in Figure 3. Within this region

are located several industrial sites in which a wealth of construction experience and of foundation and subsurface data has been obtained.

Subsurface Formations

The soil between the general level of the city (above El. 620) and the surface of the clays (about El. 560) consists of stratified sands and silts with moderate cohesion. The results of a typical boring are illustrated in Figure 4. The boring was made in Area D, Figure 3, on the slope of one of the bluffs bounding the valley. The material probably represents the original deltaic formation.

Immediately below the floor of the present valley the delta deposit appears to have been removed and replaced by soils consisting primarily of silty sands and silts interspersed with organic matter. As a rule the compressibility of these materials is relatively great and the relative density is low. This is indicated by the right-hand part of Figure 4, which represents a soil profile in Area A, Figure 3. The vertical lines correspond to borings, and the numerals on the right side of the lines represent the results of the standard penetration test. The blows per foot range from less than 1 to 10 or more, but the average value is on the order of 4. These values correspond to very loose materials. As a consequence only the most unimportant structures are established on the surface of these deposits. All major structures are established on foundations extending through the silty sands to the underlying clays.

Almost all the material below the silty sand and above the bedrock consists of glacial clays. The clays have a bluish gray color and are with few exceptions at least moderately silty. No very soft clays are encountered in the locality. In detail, the clay deposit consists of a large number of individual units each formed under its own particular geologican conditions. The characteristics of the various bodies differ to a considerable extent, and the properties of the clay in any one of the bodies are by no means constant from point to point. The general character and arrangement of the various units that make up the glacial clays can be inferred from the cross section, Figure 5.

Figure 5 is a composite drawing showing the results of three groups of borings located in Areas C, B, and A, Figure 3. Thus, the drawing does not represent a cross-section to an accurate scale but is rather an idealization intended to show the similarities and differences among the various clays.

From the four borings in Area C, continuous 2-inch thin-walled tube samples were recovered through the clay deposit of bedrock, which was encountered at about El. 450. Likewise the borings in Area A, five in number, furnished continuous 2-inch samples. These borings extended to about El. 440 but did not penetrate the deeper strata to the bedrock. In six of the borings in Area B, located between the other two areas, samples were obtained at 5-foot intervals by means of a 2-inch I.D. split spoon. In addition, one boring was made in which continuous 2-inch tube samples were obtained from El. 400 to 285. Unconfined compressive strength tests were performed on all the tube samples; the results are plotted in Figure 5. The split spoon samples as well as the tube

samples were described in detail in order to permit classification according to geological origin.

Figure 5 suggests that at least seven major elements constitute the glacial clay deposits. Some of these, indicated by the horizontal hatching, are unquestionably of lacustrine origin. They contain no pebbles and are either massive or somewhat laminated clays and silty clays. In some instances the clays contain numerous horizontal silt partings. Other materials, indicated by diagonal hatching, are unquestionably tills, deposited directly from the glacier without sorting by water. They contain pebbles of various kinds as well as inclusions of silt and sand, and show no signs of stratification. Finally, some of the materials, indicated by dashed diagonal shading, can best be classified as waterlaid or stratified tills. These tills contain pebbles and are mixed-grained materials, but they also contain occasional distinct evidence of stratification and sorting. Such soils must have been deposited when the edge of the glacial ice was at least intermittently in contact with standing water.

The index properties of each of the seven major deposits are summarized in Table 1. As would be expected, the lacustrine deposits are generally more uniform and less strong than the tills.

Both the geology of the area and the results of consolidation tests indicate that the clay deposits are all strongly preloaded. If the top of the original delta reached El. 812, corresponding to the highest lake level, the submerged weight of the material later eroded from above the present ground surface would have been about 5 tons per square foot. Consolidation tests indicate that the lacustrine deposits were compressed under a load of this magnitude, and suggest an even greater degree of preconsolidation for some of the tills.

The floor of the bedrock valley in the area shown in Figure 3 consists primarily of Chagrin shale of Ordovician age. The unconfined compressive strength of cores of the shale is generally found to be in excess of 100 tons per sq ft.

Hydrostatic Conditions

The free water surface within the valley is at or within a few feet of the level of the Cuyahoga River. Nevertheless, observations made in 1942 in Area C demonstrated that certain portions of the clay deposit are subjected to appreciable hydrostatic uplift. At the site of one large structure, several Carlson-type porewater pressure devices were established at levels of about 15, 30, and 50 ft. below the surface of the clay deposit. These were later supplemented by closed-system Bourdon-gage assemblies. As soon as the instruments were installed and before the influence of any subsequent construction could have affected the porewater pressures, the devices indicated pressures in excess of normal static pressures as shown in Figure 6. These pressures correspond to an upward hydraulic gradient of considerable magnitude through clay deposit B, Figure 5. Moreover, during the installation of some of the porewater pressure gages there was a flow of groundwater from the top of the casing when its lower end was located at about El. 510. At least

several of the borings in the area indicated the presence of sand deposits from six inches to several feet in thickness at the same level. These facts suggest that between clay deposits B and C there are patches or channels of relatively pervious granular material, possibly outwash, that communicate with the higher water table of the surrounding upland. The existence of an upward hydraulic gradient of the magnitude indicated by the porewater pressure devices is a matter of practical concern in any estimate of existing intergranular pressures in the clay deposit, and in any stability problem involving the clay.

Piles and Pile Foundations

Because of the generally unsatisfactory nature of the silty and sandy deposits just below the level of the valley proper, relatively few important structures have been founded on spread footings or rafts. Piles are the dominant type of foundation for heavy structures. They range in variety from friction piles extending a relatively short distance into the clay deposit to long point-bearing piles extending to bedrock where bedrock is at an accessible elevation. Many dozens of pile-loading tests have been performed in various parts of the area.

At location C, Figure 3, over 50 load tests were made on individual piles of various sorts. In most instances the piles were loaded to failure. The results of these tests are summarized in Figure 7 in which the compressive-strength diagrams for the deepest and shallowest borings at the site are repeated from Figure 5. Some of the piles extended only into the lacustrine deposit B. It is apparent that this layer does not possess sufficient strength and thickness to support pedestal-type piles, whereas cylindrical or slightly tapered piles developed moderate skin friction. The failure load for such piles was found to correspond to a skin friction approximately equal to the completely remolded shearing strength of the surrounding clay, in spite of the fact that the load tests were made from several days to several weeks after the piles were driven. This experience appears to be at variance with the results of pile tests in deposits of similar origin, as in Chicago and Detroit, where the skin friction appears to be approximately equal to half the unconfined compressive strength of the undisturbed clay. It is conceivable that the inability of the clay to develop its full strength adjacent to the piles is due to the failure of the clay to consolidate after pile driving on account of its preloaded character.

At location A four load tests were made on steel pipe piles with an external diameter of 10-3/4 inches. These piles extended to elevations between 465 and 483. Thus their lengths varied from about 110 to about 126 ft. The load-settlement curves for the tests are shown in Figure 8. Load-settlement curves for three similiar 10-3/4 inch piles driven in Area B are also shown in Figure 8. These piles varied from 112 to 170 ft.in length. The driving record for the longest pile is typical for the three tests. It is apparent that the piles did not derive appreciable support from end bearing.

It is noteworthy that there is no consistent relationship between the results of the load tests on piles of any type and the computed values of

the bearing capacity as determined by means of one of the common pile-driving formulas. This is to be expected in a deposit consisting essentially of clays, wherein the piles derive their support primarily from skin friction.

The driving of piles in the clay deposits has led to the conclusion that the clays are essentially imcompressible during pile driving. In the cofferdam for a bridge pier in the vicinity of Area B, Figure 3, 164 pipe piles, 10-3/4 inches in diameter, were driven to about El. 415. The spacing was 3.5 ft. The piles near the edges of the pier heaved from 1 to 5 inches, whereas those in the center heaved as much as 12 to 15 inches. It was not found feasible to redrive the piles once they had heaved. However, inasmuch as the piles derived their support primarily from skin friction, redriving did not appear necessary.

The depth to which 10-inch pipe piles can be driven with a given hammer is somewhat variable and undoubtedly reflects the nature of the subsurface conditions. A record of the length of several thousand piles driven at the site of one foundation in Area A is shown in Figure 9. It was intended that the points of the piles should reach a level between 90 and 120 feet below grade. Although most of the piles were driven to these depths, a large minority could not be forced down. The shorter piles were located in the elongated area shown in the figure. It is probable that the area represents a tongue of particularly resistant till. Although there were borings at the site on about 200-ft. centers, none happened to be located in the region of the relatively short piles. This fact demonstrates convincingly that even a conscientiously planned boring program in deposits as erratic as those in the Cuyahoga Valley cannot disclose at reasonable cost all the significant variations in subsurface conditions.

Settlement of Structures

As a general rule the settlement of ordinary structures established on adequate foundations in the upper part of the clay deposit is not excessive in the Cuyahoga Valley, on account of the relatively great degree of preconsolidation of the clays.

The only large areas now subjected to loads approaching the estimated preconsolidation load of 5 tons per sq. ft. are the storage areas for iron ore. Several of these are located in the valley. The ore is stored to a height of as much as 50 ft. Since the density of the ore is approximately 160 lbs. per cu. ft. the actual load added to the clay is on the order of 4 tons per sq. ft. The settlement of the ground beneath these storage piles appears to be somewhat less than one foot, and it is probable that the seat of this settlement is in the sand and silt above the clay. When settlement of other structures did occur, it could generally be attributed to the overloading of pile foundations enough to cause slippage of the piles with respect to the surrounding soil.

The behavior of structures of relatively large size exerting moderate loads on the original delta formations may be inferred from the history of a coke-oven battery in Area D, Figure 3. The battery was located partly in a cut made into the base of the bluff as shown in Figure 10.

Foundation conditions at the site are indicated by the first boring log in Figure 4. The structure was established upon a relatively flexible concrete mat 3 ft. thick supported by a rolled slag subbase also 3 ft. thick. The entire structure was underlain by about 40 ft. of delta deposits which, in turn, were underlain by the series of glacial clays. During excavation of the soil from the site the rise of the ground surface was measured at the points indicated in Figure 10. The excavation of 12.5 ft. of soil caused a rise of only 0.75 in. The settlements of the structure after completion, as well as the loads acting at the base of the mat, are shown in Figure 10. It is apparent that the delta deposits in the bluff are relatively satisfactory foundation materials. They are relatively pervious, however, and great care must be exercised in lowering the water table to avoid development of quick conditions during excavation.

Settlement problems have not been as significant in the valley as lateral movements. These will be discussed in the next section.

Stability Problems

Some of the most significant movements of foundations in the valley have been associated with the retaining structures built for the storage of iron ore. The lateral pressure of the ore against the retaining structures is itself very great. In addition, the weight of the stored ore produces large horizontal forces in the subsoil that tend to move the foundations of the retaining walls outward. The movement is greatest on the side of the retaining structures next to the river.

Figure 11 shows an idealized section through one such retaining structure. It also shows a summary of the movements that occurred during the first 5 years of the life of the structure. The movements tend to be progressive. Under similar loads, other ore-retaining structures in the locality have experienced movements over periods of 50 years varying from about 16 inches to 6 ft. Yet, in spite of the magnitude of these movements, there has been no outright failure of any retaining wall for an ore storage yard in the Cleveland area. The movements must all be regarded as relatively large but normal deformations.

The steel tie rods connecting the retaining walls shown in Figure 11 were utilized as strain gages to measure the horizontal load carried by the walls. This load reached a magnitude of 35 tons per lineal foot of wall when the ore reached a height of 22 ft. According to earth-pressure computations the lateral pressure of the ore at this height should have been only 8 tons per sq. ft. The rest of the pressure was caused by the weight of the ore and the consequent lateral thrust on the pile foundations of the walls. It is obvious that this supplementary pressure was of considerably greater significance than the lateral pressure of the ore itself. Many of the stability problems in the area have arisen because of failure to recognize this fact.

CONCLUSION

In this paper the general characteristics of the subsurface materials within a restricted area of the Cuyahoga River Valley have been

presented. It is obvious that the variations in the deposit are such that no conclusions should be drawn regarding subsurface conditions at a particular site on the basis of the generalized information contained herein. Nevertheless, the information in this paper provides considerable insight regarding the behavior of the subsurface materials in the valley and should serve as a basis for planning exploratory programs and and making preliminary designs on future projects.

ACKNOWLEDGMENT

The information contained in this paper has been assembled in connection with foundation projects for several private organizations with properties in the valley. Individuals especially interested in the regional foundation studies and helpful in providing data include Mr. Leonard Larson and Mr. L. T. Tanney of the Republic Steel Corp.; Mr. George S. Monks of the Jones and Laughlin Steel Corp.; and Mr. H. F. Whitmore of the W. and L. E. Railway Co. Much assistance was furnished by Mr. Linton Hart of the Raymond Concrete Pile Co. The writer acknowledges the contributions to an understanding of the regional problems by Dr. K. Terzaghi with whom he was associated on projects for the Republic Steel Corp., and data obtained by Mr. Shortridge Hardesty in connection with the construction of the W. and L. E. bridge. Substantial portions of the geological correlations were worked out by Mr. C. T. Bagley while a graduate student at the University of Illinois.

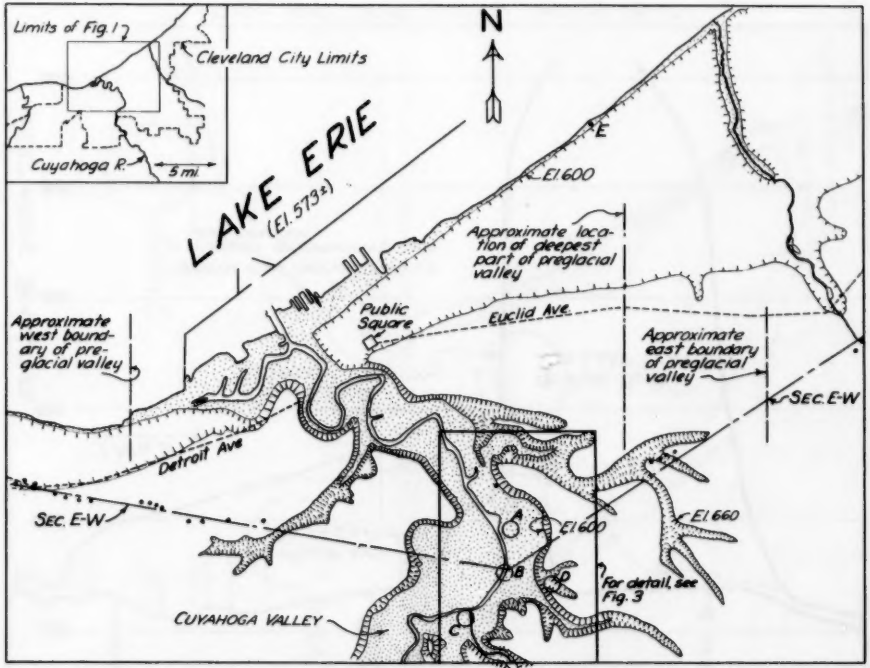

FIG. 1. LOCATION MAP OF VALLEY

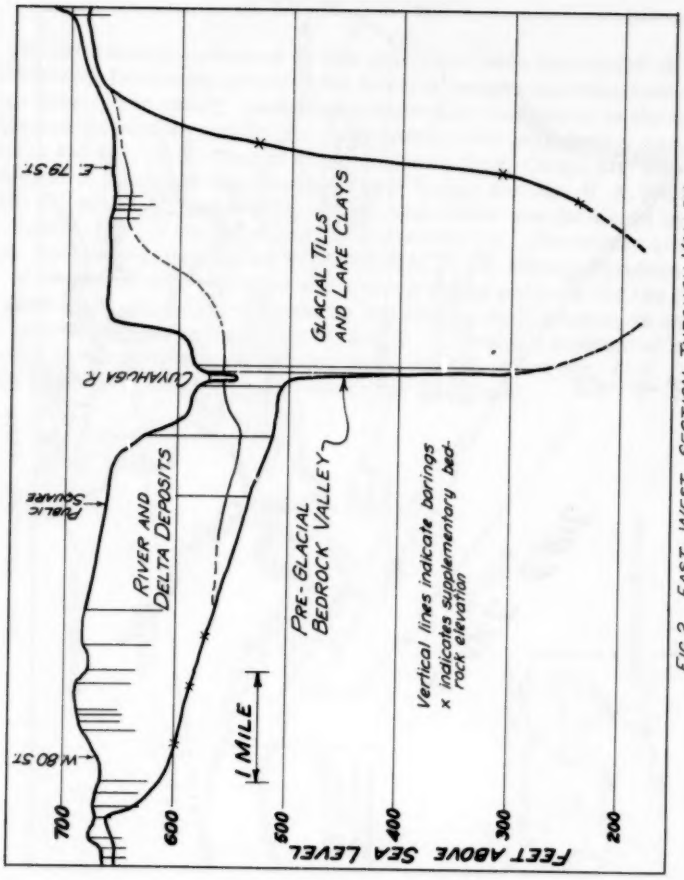

FIG. 2. EAST-WEST SECTION THROUGH VALLEY

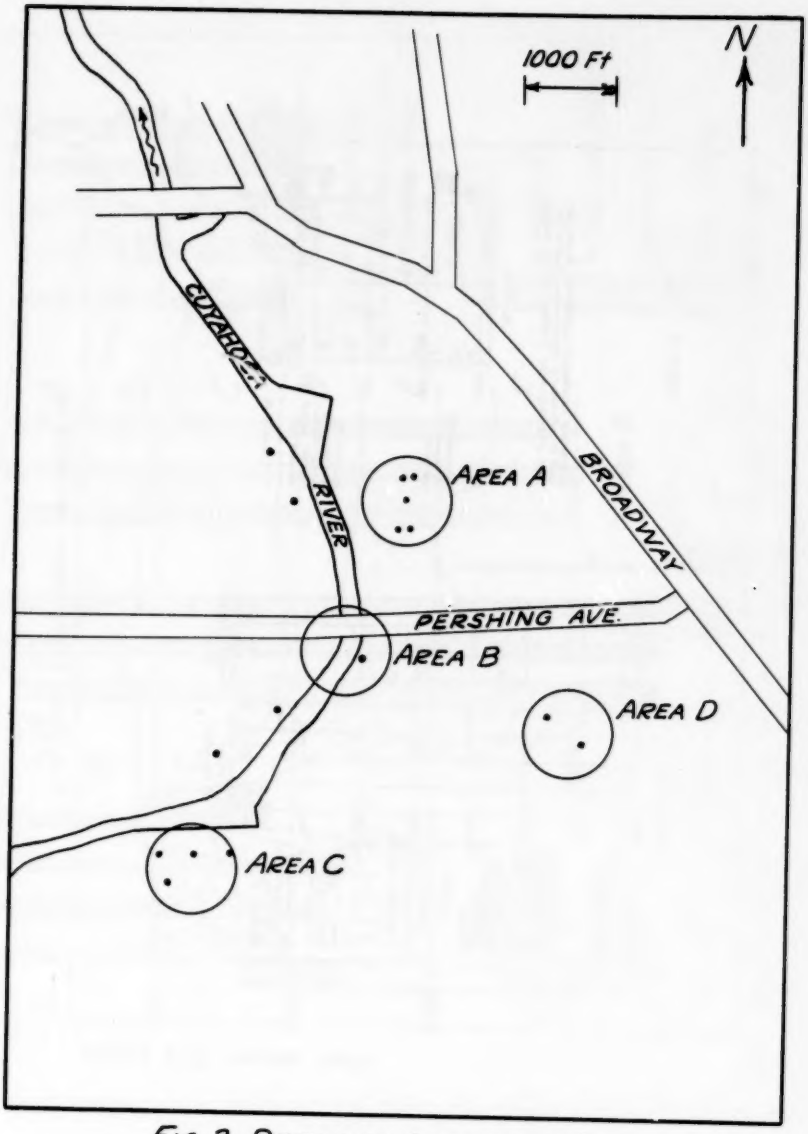

FIG. 3. DETAILED LOCATION PLAN

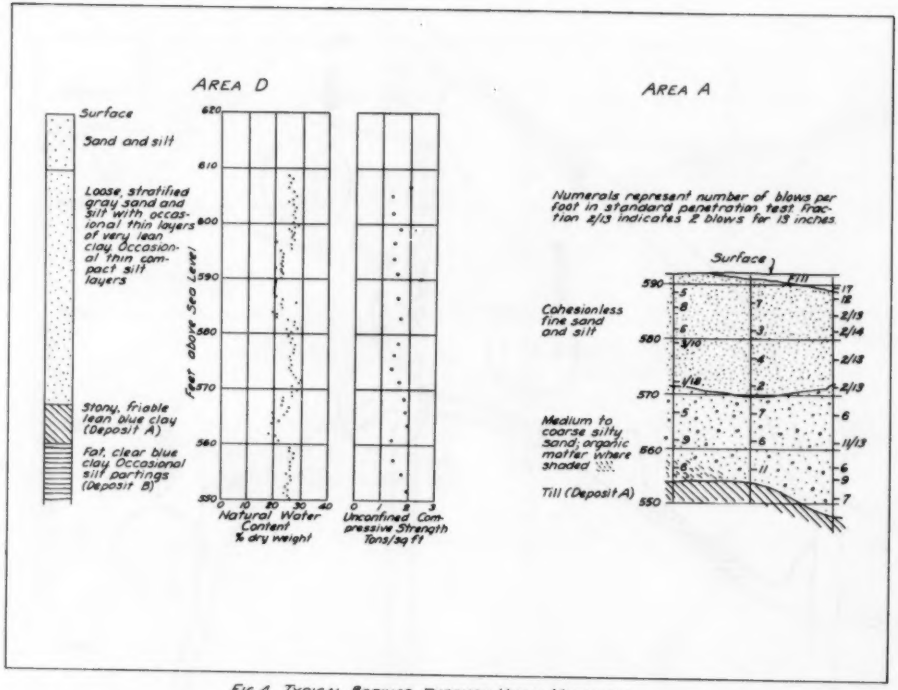

FIG. 4 TYPICAL BORINGS THROUGH UPPER MATERIALS

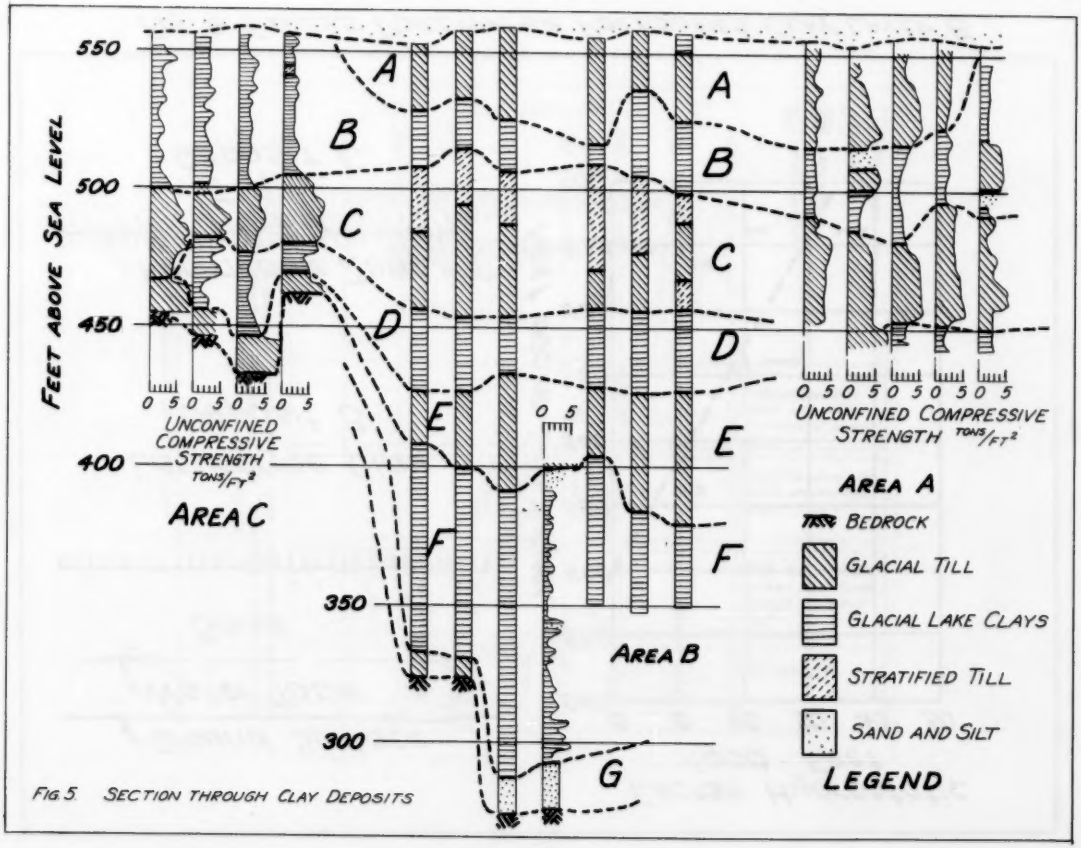

FIG. 5 SECTION THROUGH CLAY DEPOSITS

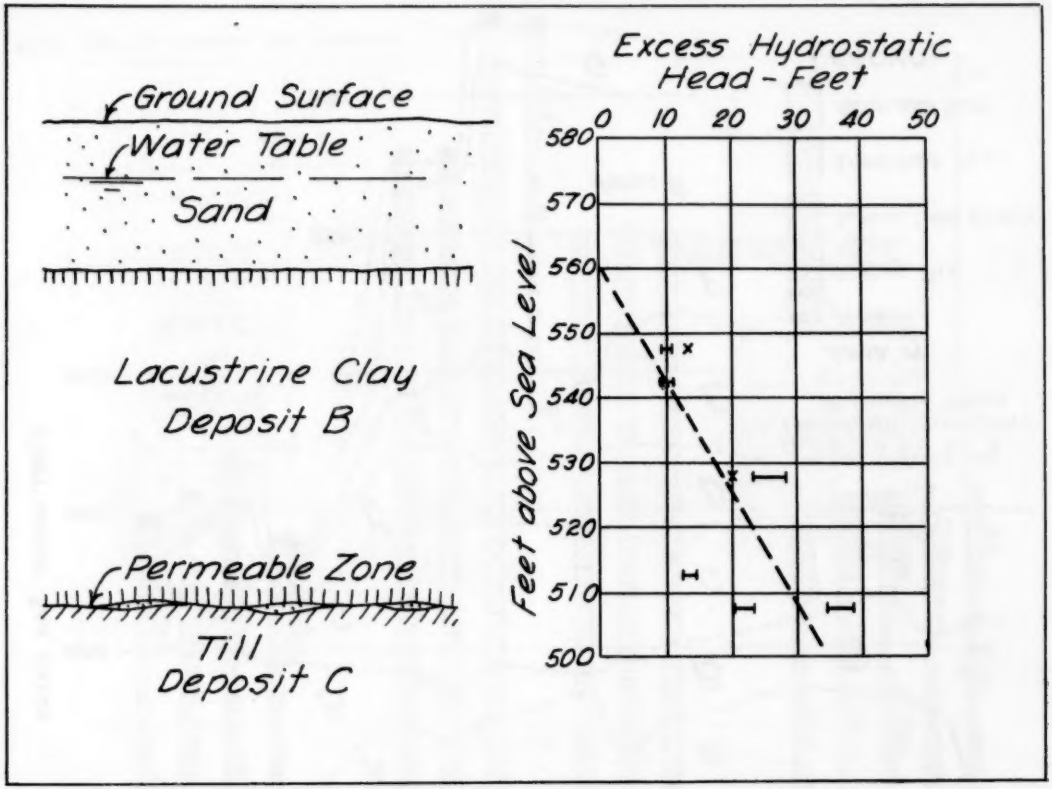

FIG. 6. EXCESS POREWATER PRESSURES, CLAY LAYER B

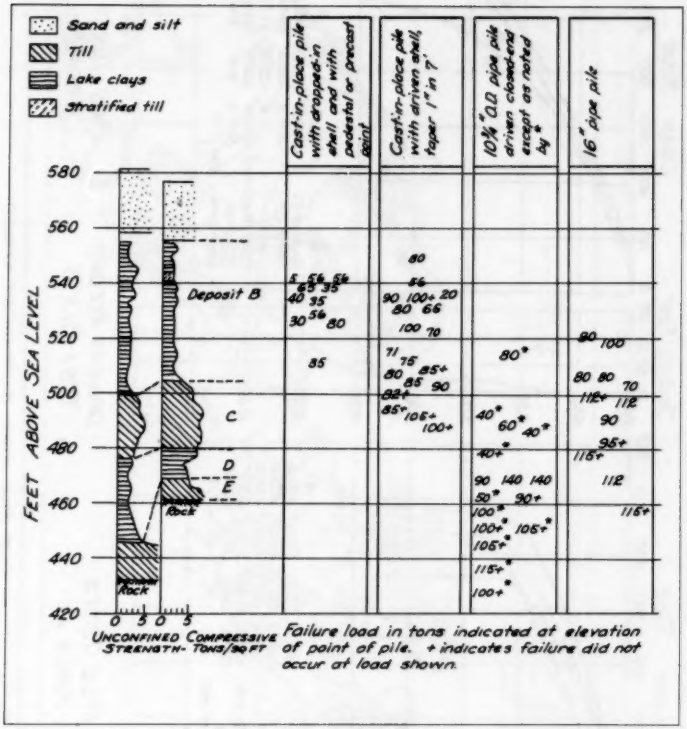

FIG. 7. SUMMARY OF PILE TESTS

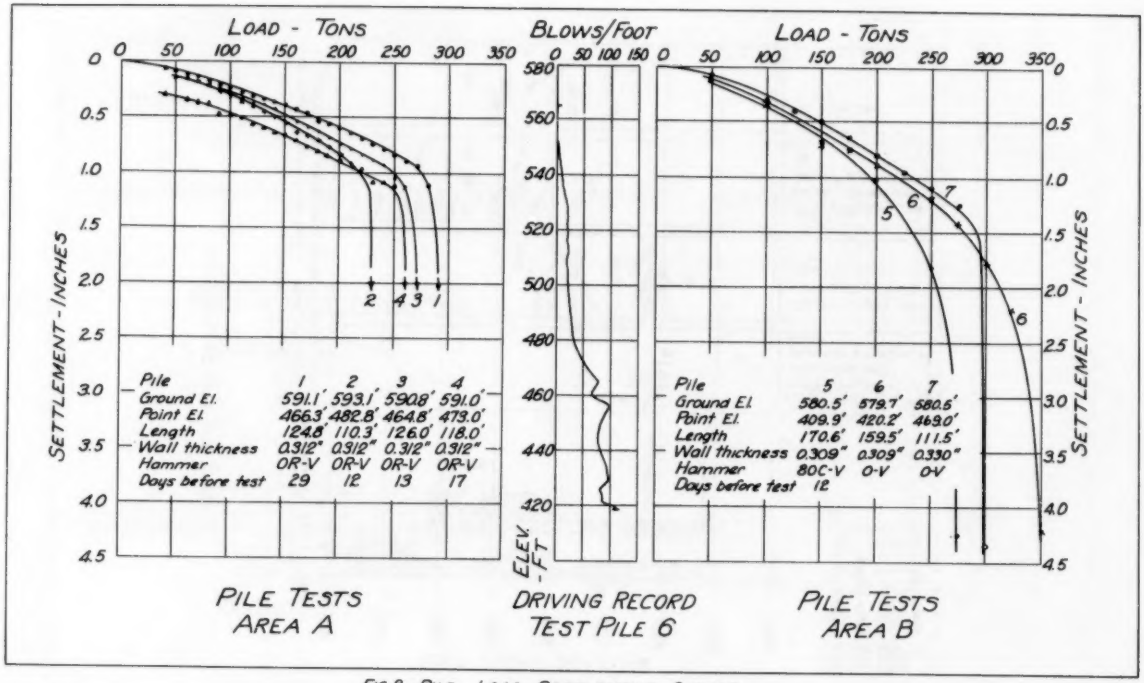

FIG. 8. PILE LOAD-SETTLEMENT CURVES

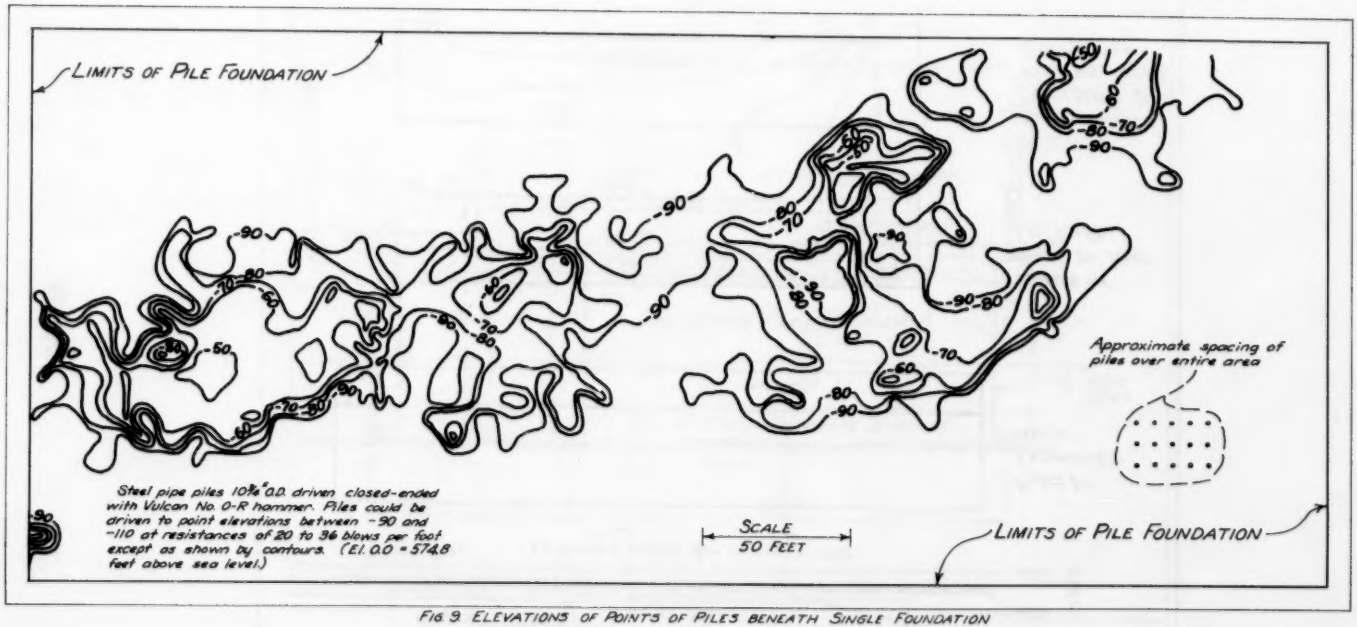
LIMITS OF PILE FOUNDATION
-50
-60
-70
-80
-90
Approximate spacing of piles over entire area
Steel pipe piles 10¾" O.D. driven closed-ended with Vulcan No. 0-R hammer. Piles could be driven to point elevations between -90 and -110 at resistances of 20 to 36 blows per foot except as shown by contours. (El. 0.0 = 574.8 feet above sea level.)
SCALE
50 FEET
LIMITS OF PILE FOUNDATION

FIG. 9. ELEVATIONS OF POINTS OF PILES BENEATH SINGLE FOUNDATION

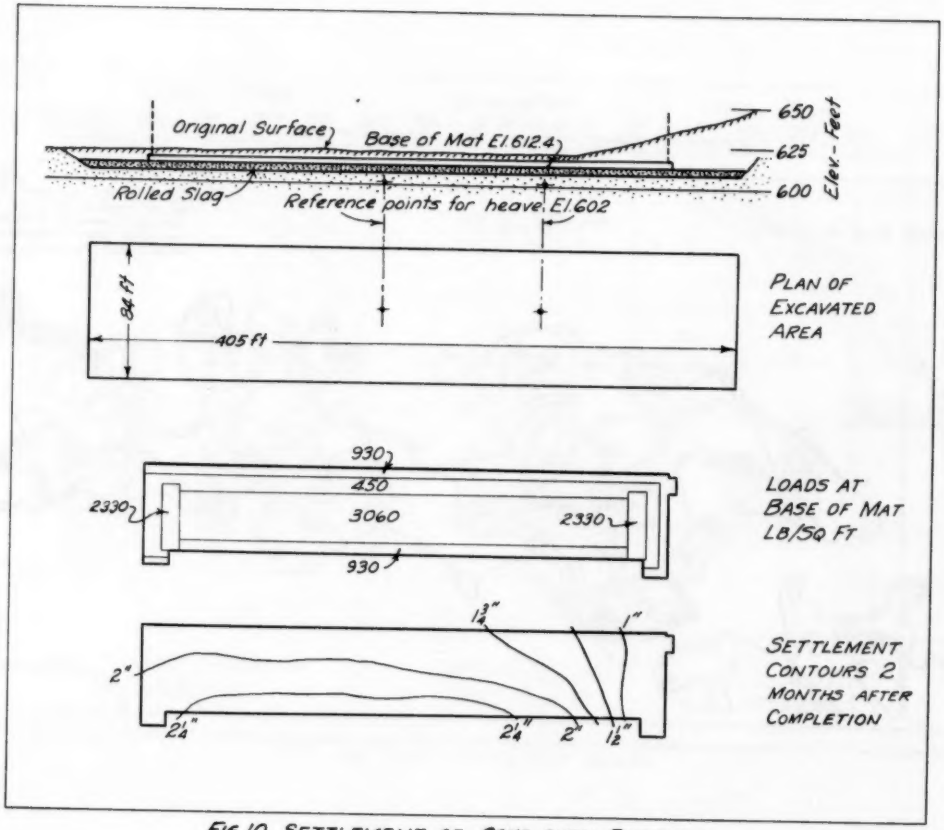

FIG. 10. SETTLEMENT OF COKE-OVEN BATTERY

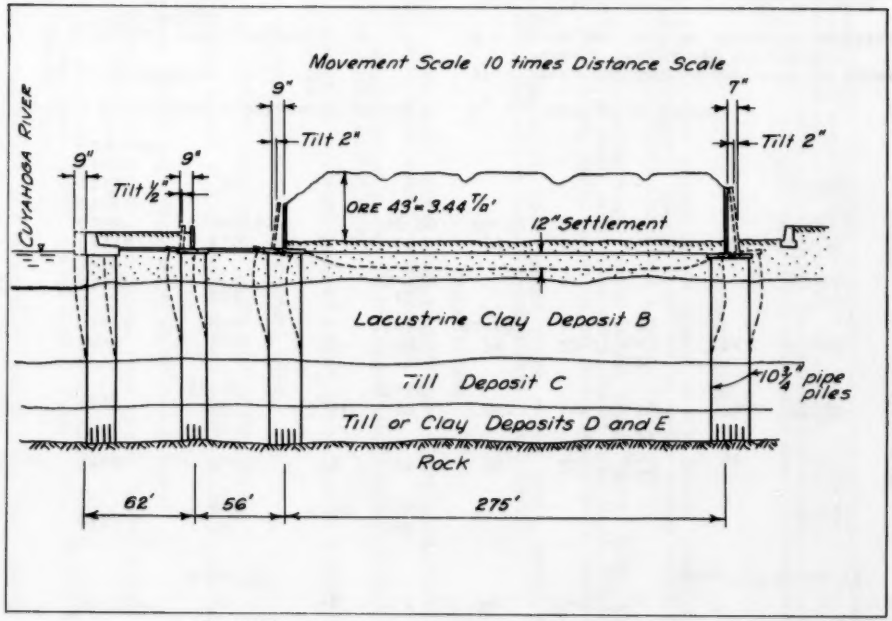

FIG. 11. MOVEMENTS OF ORE-RETAINING STRUCTURE

Table 1

TYPICAL INDEX PROPERTIES OF CLAY DEPOSITS

Deposit	Character	q_u tons/ft^2	S_t	w	L_w	P_w	C_c	p' tons/ft^2	N blows/ft	Grain Size % USBS
A	Till	3 2-5	1.5±	22 19-23					6-25	
B	Lake Clay	2.0 1.3-3.5	3±	25 23-30	36 34-38	21 20-22	0.14 to 0.31	5±	5-20	Sa 5 Si 39 C 56
C	Till	4 1.5-8	1.5±	17 15-19	28±	16±	0.14±	7±	20-40	Sa 14 Si 37 C 49
D	Lake Clay	2.5 2-6	3±	28 25-35	35 32-40	22	0.26 to 0.31	5±	10-20	Sa 2 Si 46 C 52
E	Stony Till	10± 4-15		10 8-12					30-100	
F	Silty Lake Clay	1.5 0.5-2.5	2.5±	20 16-26	30 26-32	17 16-18	0.43±	6±	20-35	Sa 10 Si 30 C 60
G	Till or Outwash	5±							40-100	

q_u = Unconfined compressive strength

S_t = Sensitivity

w = Natural water content, % dry weight

L_w = liquid limit

P_w = plastic limit

C_c = compression index

p' - preconsolidation pressure in excess of present overburden pressure

N = blows per foot in standard penetration test

USBS = Bureau of Soils classification

Average values given first. Usual range in values follows.

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NOVEMBER: 321(ST), 322(ST), 323(SM), 324(SM), 325(SM), 326(SM), 327(SM), 328(SM), 329(HW), 330(EM)[a], 331(EM)[a], 332(EM)[a], 333(EM)[c], 334(EM), 335(SA), 336(SA), 337(SA), 338(SA), 339(SA), 340(SA), 341(SA), 342(CO), 343(ST), 344(ST), 345(ST), 346(IR), 347(IR), 348(CO), 349(ST), 350(HW), 351(HW), 352(SA), 353(SU), 354(HY), 355(PO), 356(CO), 357(HW), 358(HY).

DECEMBER: 359(AT), 360(SM), 361(HY), 362(HY), 363(SM), 364(HY), 365(HY), 366(HY), 367(SU)[c], 368(WW)[c], 369(IR), 370(AT)[c], 371(SM)[c], 372(CO)[c], 373(ST)[c], 374(EM)[c], 375(EM), 376(EM), 377(SA)[c], 378(PO)[c].

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a. Presented at the New York (N.Y.) Convention of the Society in October, 1953.
b. Beginning with "Proceedings-Separate No. 290," published in October, 1953, an automatic distribution of papers was inaugurated, as outlined in "Civil Engineering," June, 1953, page 66.
c. Discussion of several papers, grouped by Divisions.
d. Presented at the Atlanta (Ga.) Convention of the Society in February, 1954.
e. Presented at the Atlantic City (N.J.) Convention in June, 1954.

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